

# DESIGN – TECHNICAL NOTE

JBA Project Code 2014s1358  
Contract Isle of Man Sea Defence Options  
Client Department of Infrastructure, Isle of Man Government  
Day, Date and Time 26/08/2014  
Author Alec Dane  
Subject Peel Open Coast – Option POC1 – Rock Revetment



Project Title: Isle of Man Sea Defence Options			Sheet No: 1
Subject: Peel Open Coast – Option POC1 – Rock Revetment			Calc No:
Job No: 2014s1358			Version:1.0
Developed By: Alec Dane	Date: 26/08/2014	Revised By: Alec Dane	Date: 11/12/2014
Checked By: Graham Kenn	Date: 09/12/2014	Approved By: Graham Kenn	Date: 11/12/2014

## 1 Aim

JBA have been commissioned by the Department of Infrastructure (DoI) to develop a number of technically viable solutions to address the still water level flooding in harbour environments and wave overtopping in open coast environments, at seven coastal sites across the Isle of Man.

This technical note covers the design assumptions, decision making process and methodology for the concept design of Option POC1, an option to place a rock revetment in front of the existing sea defences in Peel, to reduce wave overtopping to the hinterland.

The scope of works does not include a formal options appraisal process. However a high level Multi Criteria Analysis will be undertaken with input from key stakeholders to help determine which option best satisfies the project criteria. The option proposed has been developed based on technical feasibility, engineering judgement, environmental impact, cost and consideration of the long term vision and key criteria determined by the project stakeholders.

## 2 Assumptions

The following assumptions have been used during the development of the concept design.

### 2.1 Datum

All elevation and depth measurements presented in the conceptual design of defence options will be presented in Douglas02 datum.

### 2.2 Baseline conditions

The open coast defences to the east of Peel are frequently overtopped by waves during a storm event. In order to design an option that efficiently reduces the risk of wave overtopping damage to the hinterland, it is important to look at the baseline conditions.

Wave overtopping occurs where the waves run up the face of the coastal defence. Where this run up exceeds the defence crest level, water passes over the structure and inundates the land behind. This design option will therefore seek to reduce the volume of flood water travelling over the existing defences during a storm event.

#### 2.2.1 Existing defence geometry

The existing defences are composed of a large block work sea wall fronted by a small amount of rock armour scour protection. During a high tide, a large water depth is in contact with the vertical sea wall, allowing large waves to impinge on the structure. To the west of the site, a sandy beach has built up, reducing the proportion of the vertical wall face in direct contact with wave action. This has the effect of reducing the wave overtopping experienced in the western section. Consequently this design option seeks to reduce the wave overtopping experienced in the eastern section.

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Attribute	Dimension
Sea wall crest level	5.6mD02
Rock armour level	2.3mD02
Sea wall height	3.3m
Rock toe level	-0.6mD02

Table 2-1: Existing defence geometry summary

## 2.2.2 Current wave overtopping risk

Based on baseline modelling of the existing defences, the eastern section of Peel is currently offered a 1 in 20-year level of protection against wave overtopping. However, by including an allowance for climate change up to the year 2115, that standard of protection reduces to less than a 1 in 1-year. This highlights the requirements for defence improvements, to provide protection to Peel promenade and the adjacent property.

## 2.2.3 Current still water level flood risk

Based on the predicted extreme water levels from the Environment Agency Coastal flood boundary conditions for UK mainland and islands project<sup>1</sup>, a maximum SWL of 4.42mD02 for the 1 in 200-year event including an allowance for climate change is predicted. Based on this prediction, it is considered that there will be no risk of flooding to the town caused solely by static water / tide levels over the open coast defences, as the primary defence has a crest elevation of 5.60mD02.

## 2.3 Design life and level of protection

The structure has been designed to achieve the following design standards:

- **Design life:** 100 years
- **Design storm event:** 1 in 200-year event (including climate change)

## 2.4 Climate change

By selecting a design life of 100 years, it is important to factor in the predicted effects of climate change. The latest UK Climate Projections (UKCP09) have been used to determine climate change allowance for:

- Still water levels;
- Wind driven waves; and
- Swell waves.

Within UKCP09 estimates for sea level rise are provided under low, medium and high emissions scenarios. Within the three scenarios the estimate is further refined by 5<sup>th</sup>, 50<sup>th</sup> and 95<sup>th</sup> percentile confidence ratings. In simple terms this should be interpreted as the relative likelihood of the projected change being at, or less than, the given change. For this study it is proposed that the medium emissions scenario is considered and that the 95<sup>th</sup> percentile confidence rating is used. This gives a projected sea level rise of 640mm by the year 2115 for Peel.

UKCP09 acknowledges the difficulty in predicting changes in wind speeds over the next 100 years and concludes that there will be a negligible increase in wind speed. Therefore, the wind driven wave component of the numerical modelling has no direct increase in wave intensity due to climate change. However, as a result of the increased still water levels from relative sea level rise, there will be an indirect increase in wind driven wave height. As a result of the larger depth of water at the coastal defence toe, larger waves will be able to travel inshore before breaking, creating a higher intensity wave climate in the year 2115.

For changes in swell waves, UKCP09 gives a prediction of the change in annual maximum wave height for the year 2115 of up to **1.0m** for the UK. It should be noted that wave height increases could be limited by the water depth at the study location and therefore the full 1.0m increase is not applicable for all scenarios.

<sup>1</sup> Coastal flood boundary conditions for UK mainland and islands, Project: SC060064/TR2: Design sea-levels. Environment Agency, Feb 2011.

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The 1.0m allowance has therefore been applied to offshore swell wave conditions, which was subjected to wave transformation modelling to determine the change in wave height at each individual site.

## 2.5 Hydrodynamic data

The hydrodynamic data, used to design the open coast defences to a 1 in 200-year standard of protection in 2115, has been sourced from three primary sources:

1. **Extreme sea levels** - The Environment Agency Coastal flood boundary conditions for UK mainland and islands project<sup>2</sup>, which developed a consistent set of design sea levels for Scotland, England and Wales.
2. **Extreme winds** – Calculated using established methods in BS6399
3. **Extreme swell waves** - The extreme wave conditions were adopted based on the Environment Agency's *Coastal flood boundary conditions for UK mainland and islands* project<sup>3</sup> which developed a consistent set of design swell wave conditions around Scotland, England and Wales.

These three sources of data were combined using joint probability analysis to create the hydrodynamic input conditions for the design of these defences for any given return period.

## 2.6 Performance standards

For coastal defences, the performance standards can typically be split into two areas, the still water level performance and wave overtopping performance.

### 2.6.1 Performance standard 1 – still water level flood risk

As discussed above, the current defences are offering in excess of a 1 in 200-year level of protection in 2115 against still water level flooding. Hence this design option will not seek to raise the impermeable defence level to address still water level flood risk.

### 2.6.2 Performance standard 2 – wave overtopping risk

Two thresholds have been used to limit the volume of overtopping that is deemed acceptable for the concept design options:

1. The first lower threshold was established for a common coastal storm event, considered to have a 1 in 1-year return period, based on a joint probability assessment.
2. The second higher threshold will be established for the design storm event, considered to have a 1 in 200-year return period, based on a joint probability assessment. During this event it is considered that general public use of the pavement and road immediately behind the structure will be discouraged and only trained personnel will be operating within the vicinity of the structure.

Table 2-2 below summarises the guidance for vehicles and pedestrians provided within the European Wave Overtopping Manual (EurOtop).

Table 2-2: Limits for overtopping for vehicles (source: EurOtop<sup>4</sup>)

Hazard type and reason	Mean discharge	Max volume
	q (l/s/m)	V <sub>max</sub> (l/m)
Driving at low speed, overtopping by pulsating flows at low flow depths, no falling jets, vehicle not immersed.	10 - 50 <sup>5</sup>	100 – 1,000
Driving at moderate or high speed,	0.01 – 0.05 <sup>6</sup>	5 – 50 at high level or

<sup>2</sup> Coastal flood boundary conditions for UK mainland and islands, Project: SC060064/TR2: Design sea-levels. Environment Agency, Feb 2011.

<sup>3</sup> Coastal flood boundary conditions for UK mainland and islands, Project: SC060064/TR3: Design swell-waves. Environment Agency, Feb 2011.

<sup>4</sup> Pullen, T., Allsop, W., Bruce, T., Kortenhaus, A., Schuttrumpf, H & van der Meer, J (2007) 'Wave overtopping of sea defences and related structure: Assessment manual'. Accessed from [www.overtopping-manual.com](http://www.overtopping-manual.com)

<sup>5</sup> Note: These limits relate to overtopping defined at highways.

<sup>6</sup> Note: These limits relate to overtopping defined at the defence, assumes the highway is immediately behind

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impulsive overtopping giving falling or high velocity jets.		velocity
Trained staff, well shod and protected, expecting to get wet, overtopping flows at lower levels only, no falling jet, low danger of fall from walkway	1-10	500 at low level
Aware pedestrian, clear view of the sea, not easily upset or frightened, able to tolerate getting wet, wider walkway	0.1	20-50 at high level or velocity

The following twofold tolerable discharge thresholds have been proposed for all concept options on open coast environments:

- 1 in 1-year event –  $<0.1\text{l/s/m}$
- 1 in 200-year event –  $<10\text{l/s/m}$ .

These tolerable discharges are such that all structures will be considered safe for pedestrian access during the more regular storm event, while vehicular and emergency staff will be safe to inspect defences during the less frequent, higher magnitude storm.

By adopting a twofold approach to acceptable overtopping levels, the new defence options considered for the sites have a dual purpose of preventing the frequent overtopping caused by common storms while providing structural and overtopping protection during rare events. By incorporating dual overtopping targets the crest height of all structures can be minimised, reducing both construction cost and visual impact.

## 2.7 Ground conditions

No geotechnical or ground condition information has been made available as part of this study. Therefore, all designs of defence structures have been progressed assuming poor ground conditions e.g. low bearing capacity. This should provide a conservative approach to the development of the concept design. The levels presented in the drawings represent finished defence levels, so would require consideration of potential settlement which would be taken into account during detailed design.

## 2.8 Structural design

A full structural design has not been included within this study as the scope of works did not include geotechnical investigation or analysis. All designs will be reviewed by a structural engineer to confirm that the design principles adopted are acceptable.

## 2.9 Services information

No detailed services information was provided as part of this study and a services search is not included within the scope of works. However, the location of more critical services has been identified by DoI. These critical services were considered in the development of the concept design options. If the project progresses to outline and detailed design it will be essential that a full service plan is developed.

## 2.10 Environmental impact

This commission does not include any formal Environmental Impact Assessment or Landscape Visual Impact Assessment. If the project progresses to outline and detailed design, a more in depth study of the environmental impacts will be required.

## 2.11 Reinstatement and finish details

The development of landscape and architectural enhancements are outside the current project scope of works. It is assumed that following construction the surrounding area will be re-instated to a condition similar to the present. However, during the detailed design stage further architectural and landscape enhancements could be considered.

## 2.12 Contaminated land

No information regarding the location of areas of contaminated land has been provided as part of this

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commission. Therefore all design options have been developed with the assumption that none of the areas are subject to contaminated land constraints. An invasive contaminated land survey should be undertaken at all locations prior to detailed design to enable detailed assessment of suitable construction techniques and options for removal or re-use of excavated material.

To progress concept design options as part of this study the following have been assumed:

- No investigation of contamination issues at individual development sites; and
- Development flood defence options may require some contaminated land treatment depending on the result of the investigations.

## 2.13 Tie in details

Tie-in details between old and new defences have been considered at a conceptual level. The key consideration has been to develop an option that does not create an area of outflanking or weak point, where overtopped water can bypass the defences and flood the hinterland. Careful consideration of the connection between the existing and new defences will be required during the detailed design phase.

## 3 Standards, guidance & reference documents

All design assumptions have been developed using the following reference material:

- BS 6180 1999: Barriers in and about buildings, code of practice
- BS EN 206-1:2000 Concrete – Part 1: Specification, performance, production and conformity
- BS EN 12620:2002 Aggregates for concrete
- BS EN 6349-1-1:2013, Maritime works, General, Code of practise for planning and design.
- CIRIA (2010), The use of concrete in maritime engineering – a guide to good practise
- Cobb, F (2009), Structural Engineers Pocket Book (2<sup>nd</sup> Edition)
- DEFRA (2009) UK Climate Projections 09
- Environment Agency. (2010). Fluvial Design Guide
- HR Wallingford (2007), EurOtop, Wave Overtopping of Sea Defences and Related Structures: Assessment Manual
- US Army Corp of Engineers (2002), Coastal Engineering Manual

## 4 Design development

The following provides a brief summary of how the key design elements were selected.

### 4.1 General form of defence

This design option acts to dissipate the wave energy arriving at the coastline by placing a permeable structure in front of the existing sea wall. This has been achieved through the design of a rock armour revetment. By placing a rock armour revetment in front of the large sea wall, the depth of water in direct contact with the sea wall has been significantly reduced. This acts to reduce the impulsive wave overtopping experienced at the coastal defence.

#### 4.1.1 Defence crest level, crest width and slope

A defence crest level of 5.10mD02 has been proposed for the rock armour revetment. This has been defined by an iterative process using the EurOtop overtopping tool and engineering judgement. The initial design of this revetment considered a full suite of revetment crest elevation, widths and slopes which were all tested against a range of wave height and water level combinations that comprises a 1 in 200-year event including an allowance for climate change and a 1 in 1-year event again including an allowance for climate change. The primary aim of this modelling was to determine the worst case combination for anticipated overtopping volumes. This initial modelling provided a guide for the general defence geometry appropriate for this location. In this case, the greatest reduction in overtopping rates were seen for a crest elevation equal to 0.5m below the current sea wall height with the front slope, sloping a 1:2.

The Design Input Statement set out limits for overtopping and are again presented here, <0.1 l/m/s for a 1

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in 1 year event and <10 l/m/s for a 1 in 200 year event. Using the iterative overtopping process, defence crest widths were varied at 100mm increments to identify the required width of crest to achieve these tolerable overtopping limits. This iterative process showed that a crest width of 12m was sufficient to reduce the overtopping to those set out above.

Optimisation of revetment crest width (1 in 200-year storm)

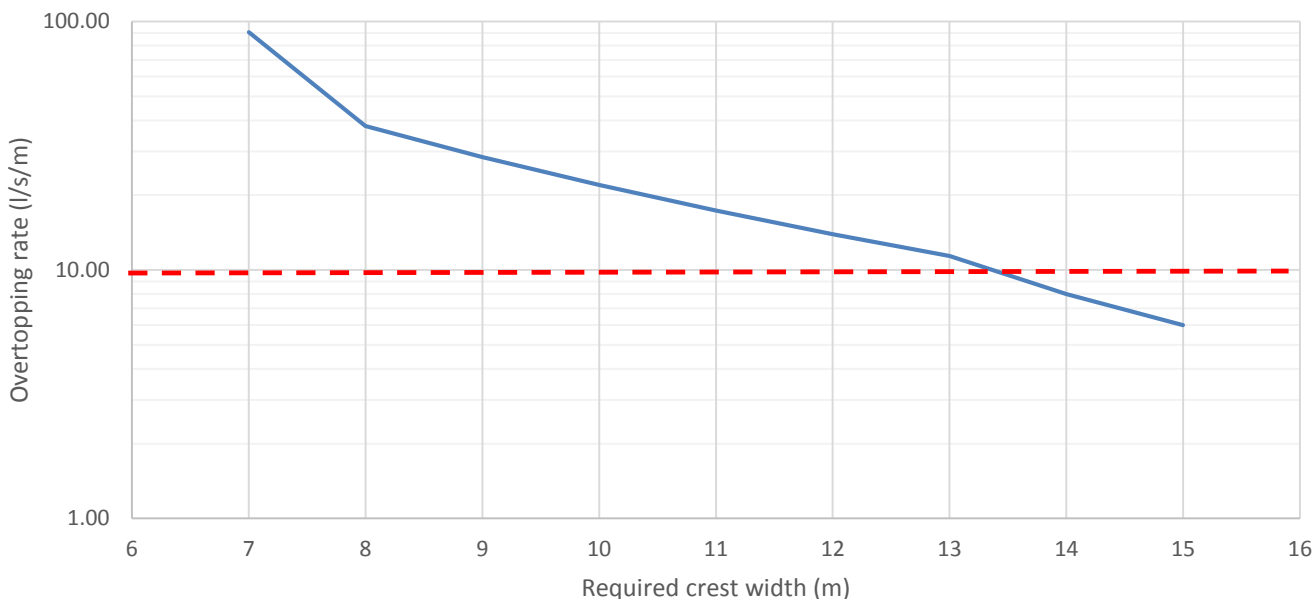


Figure 4-1: Optimisation of POC1 crest width (1 in 200-year storm)

	Threshold Value	Value
<b>Crest Elevation (mD02)</b>	-	5.10
<b>Crest Width (m)</b>		12.00
<b>Overtopping Rate 1:1 (l/m/s)</b>	0.1	2.29
<b>Overtopping Rate 1:200 (l/m/s)</b>	10	11.67

Table 4.1: Defence configuration and overtopping rates for POC1

For the 1 in 200-year event, an overtopping rate of 11l/s/m is quoted. While this is not less than the required threshold, given the accuracy of the overtopping model (orders of magnitude), this is deemed to have made the threshold. It should also be noted, that this defence configuration does not conform to the tolerable threshold for the more regular 1 in 1-year storm (<0.1l/s/m). In order to meet this threshold, the crest width would have to be in the order of 18m which would be a considerable capital scheme. An overtopping rate of 2l/s/m is suggested as being acceptable for a well shod, trained member of staff to access the land behind<sup>7</sup>. Given eastern Peel’s relatively open promenade, the DoI could consider this overtopping threshold acceptable for the more regular storm event. However, the lack of conformity to the design standard should be noted and considered when evaluating other options in the appraisal. It is important that the DoI understand the implications of accepting this higher overtopping rate and the need to plan for area cordoning during the storm event.

It should be noted that the EurOtop guidance suggests that the model is only suitable for the development of concept design options. Physical modelling is recommended for detailed design stages, if control of overtopping volumes forms one of the key design criteria.

<sup>7</sup> Pullen, T., Allsop, W., Bruce, T., Kortenhaus, A., Schuttrumpf, H & van der Meer, J (2007) 'Wave overtopping of sea defences and related structure: Assessment manual'. Accessed from [www.overtopping-manual.com](http://www.overtopping-manual.com)

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## 4.2 Rock armour sizing

The following summarises the primary rock armour design process (further details can be found within the calculation sheet). The rock armour has been sized using the ultimate limit state, or the upper limit for the structural stability of the proposed defence components. This limit state has been used to ensure the rock armour units will withstand 200-year wave conditions in combination with 200-year extreme sea-levels, including the effects of climate change to the year 2115. The overall likelihood of an event of this magnitude occurring will have a probability greater than 200 years, incorporating a preliminary level of safety into the critical design elements. This has ensured any structures will withstand wave conditions with an extremely low recurrence interval.

- Rock armour sizing calculation used: Van der Meer & Hudson
- Significant wave height ( $H_s$ ): 1 in 200 year ultimate limit state wave height plus climate change transformed to the structure toe = 2.90m
- Period ( $T_m$ ): 1 in 200 year plus climate change wave period = 6.66s
- Still Water Level (SWL): 1 in 200 year water level including climate change = 5.62mD02
- Permeability Factor (P). HR Wallingford (1998) Revetment systems against wave attack - A design manual (page 89). The proposed structure will be formed with a minimum of 2 layers of armour stone placed directly on a relatively impermeable core, therefore  $P=0.1$  has been selected as an appropriate permeability factor.
- Slope Angle ( $\alpha$ ): 1 in 2 ( $27^\circ$ ) has been selected to represent a shallow slope while attempting to limit the total required volume of rock
- Damage Number ( $S_d$ ). HR Wallingford Revetment Systems Against Wave Attack - A Design Manual (page 89) states: *'For most cases, design damage is set at  $S_d=2$  as equivalent to the "no damage" limit'*.
- Storm duration was set at 4 hours to cover two hours before and after high tide.

Using the above input parameters within the Van der Meer calculation provides a median required rock mass ( $M_{50}$ ) = **2.692t**. Therefore adopting a conservative approach by selecting a standard rock grading above the median predicted rock size results in a standard rock grading of **3 - 6 tonnes** being selected for the armour stone. Figure 3 shows that less than 5% of the rock in this grading will be smaller than the required  $M_{50}$  and so represents a suitable material grade for the environment. The revetment, therefore has a designed median required rock mass ( $M_{50,des}$ ) = **4.50t** and a designed median required rock diameter ( $D_{n50,des}$ ) = **1.19m**.

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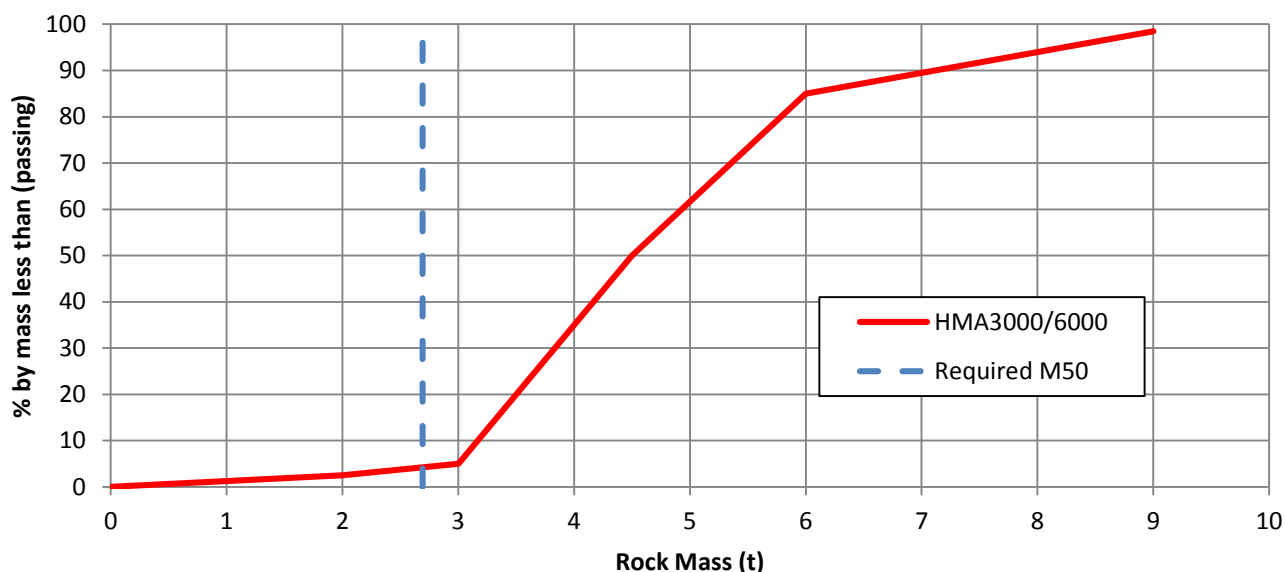


Figure 4-1: Grading curve for HMA 3000/6000 and the required M50

## 4.2.1 Filter layer

The permeability of the structure has a direct impact on the structural stability of the armour units. With a greater permeability, wave energy can be dissipated through a larger armour unit depth and reduce the development of interstitial pore pressures that can result in the expulsion of armour units. In order to increase the permeability, it is essential to provide an adequate filter layer of a minimum  $1.5D_{n50,a}$  thickness and a  $D_{n50,f} = D_{n50,a}/2.35$ . Using these formula, the filter layer has a minimum thickness ( $t_{d,f}$ ) of **1.8m**, a nominal median required rock mass ( $M_{n50,f}$ ) = **0.34 t** and a nominal median required rock diameter ( $D_{n50,f}$ ) = **0.50m**. Selecting a standard rock grading above the median required mass results in a distribution of **0.3-1t**. Note: a rock grading above the median required rock size has been selected for the filter layer as the stability of this layer is not dependent on the mass but upon the permeability. It is therefore, important to select the grading above the required median diameter.

## 4.2.2 Core

The design specifies a semi-permeable core to allow some dissipation of energy through the core. Suitably graded material should be selected with both the required sizes and economic feasibility in mind. Where it may be suitable to place a more impermeable core, the depth of the filter layer may be increased to allow for a reduction in the permeability. These options should be considered during the detailed design phase. The core should be adequately compacted into the existing armour units to minimise the risks of subsidence.

## 4.3 Structure toe and foundation level

The rock revetment has been designed to be placed directly on to the existing rock platform to reduce the risk of scour at the structure toe.

The stability of the leading edge of the toe armour is likely to be reduced due to reduced friction between interlocking layers (Environment Agency, 2013). These rocks are more vulnerable to movement due to the absence of support from the seaward side. In order to mitigate this potential failure mechanism, the design specifies using the largest rocks as the keystone (a minimum of 4.5 tonnes).



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## 4.4 Packing density

The packing density of the armour layer has a direct impact on the performance of the structure as well as the total volume occupied. The following assumptions have been made in calculating the packing density and total required volumes for the defence configuration outlined above:

- Packing density calculation used - CIRIA -The Rock Manual - The Use of Rock In Hydraulic Engineering (page 124).
- Chainage of total length to protect ~440m
- Volumetric layer porosity ( $n_v$ ). CIRIA - The Rock Manual - The Use of Rock In Hydraulic Engineering (page 126) states that: *'for a double layer of irregular rock placed in standard packing, a value of 32% should be used for the volumetric layer porosity'*.
- Layer thickness coefficient ( $k_t$ ). CIRIA - The Rock Manual - The Use of Rock In Hydraulic Engineering (page 126) states that: *'for a double layer of irregular rock placed in standard packing, a coefficient of 0.87 should be used for the layer thickness coefficient'*.
- The rock is assumed to have a porosity ( $p$ ) of 0.1.
- The degree of saturation ( $S_r$ ). CIRIA - The Rock Manual - The Use of Rock in Hydraulic Engineering (page 97) states that: *'for an armour stone that is not in permanent contact with water, a saturation of 0.25 should be used'*.

Using the above input parameters, the following packing densities and total required volumes have been calculated. In addition, the previous calculations have been summarised, to provide a lookup table for the rock requirements. It should be noted that the total required masses have been calculated based on a constant foundation depth of -1.15mD02 which is known to vary considerably. Consequently, the actual required mass may be significantly higher or lower than the values calculated here.

Required Median Mass ( $M_{n50,a,req}$ )	2.69t
Designed Median Mass ( $M_{n50,a,des}$ )	3-6t (4.5t median)
Required Median Diameter ( $D_{n50,a,req}$ )	1.00m
Designed Median Diameter ( $D_{n50,a,des}$ )	1.19m
Thickness of Layer ( $t_{d,a}$ )	2.06m
Packing Density ( $\rho_{b,a}$ )	1.77t/m <sup>3</sup>
Cross Sectional area of Structure ( $A_a$ )	81m <sup>2</sup>
Total Required Rock Mass ( $W_{b,a}$ )	68,000t

Table 4-2: Rock sizes and estimated requirements

It should be noted that these represent estimated material requirements and should not be used for ordering materials.

## 4.5 Tie in details

It is proposed that the structure will terminate at the wider beach to the west of the section. The exact location of this tie in cannot be confirmed at this design stage. During detailed design, it is recommended that multiple sections are analysed to identify the exact location of the defence tie in. To the east of the site, it is anticipated that the rock armour revetment will tie into the rock cliff to prevent an area of weakness developing.

## 4.6 Access for the public

By providing a physical barrier between the promenade and the beach, it is necessary to provide designated access routes for the public to access either side. This access could be achieved through placing stainless steel access steps at strategic locations. The best method for achieving this access should be developed during a latter design stage.

## 4.7 Public safety

Public safety has formed a key consideration during the concept design development phase. The main risks associated with this option are the issues surrounding the future public usage of the structure. The

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public should be discouraged from climbing on the rock structure as there is a risk of injury. It is suggested that signage is used as a means of warning. In addition, it is recommended that stainless steel access steps are provided over the defence, to allow members of the beach to access either side of the structure.

In addition, by offering a lower standard of protection or allowing a higher tolerable threshold during the regular storm event, the risk of public interaction with water overtopping the defence is higher than if it conformed with the intended design standards. If this option is progressed, the DoI should implement a storm action plan to control these risks, to prevent pedestrians encountering the overtopped water.

For further information on all the risks considered, mitigated or reduced please refer to the Designers Hazard Inventory.

## 5 Technical risks summary

The following are considered to represent the key risks highlighted during the development of this concept design.

### 5.1 Unknown ground conditions

Due to the unknown ground conditions it is possible that the current design will require modification in order to achieve structural and geotechnical stability.

### 5.2 Integrity of the sea wall

While the seawall face seems to be in reasonable condition, the placement of rock armour adjacent to the existing structure represents a technical risk. Care should be taken to not drive plant on the landward side of the wall, minimise any required excavations and take care when rock is placed in direct contact with the existing defences. All attempts to prevent loading to the sea wall should be made.

The 100 year design life of the coastal defence is dependent on the structural integrity of the existing defences, as this new structure forms part of a composite coastal defence. This design assumes that the existing sea wall will not be allowed to deteriorate further as this may undermine the newly proposed superstructure. It is recommended that a full asset inspection be undertaken prior to detailed design, to quantify the residual life of the structure and allow for the development of more tailored remediation measures.

### 5.3 Interface with existing armour

To form the new structure, the quarry run core should be adequately compacted over the existing armour units. If an adequate compaction is not achieved, there is the risk of subsidence which could reduce the crest elevation of the primary armour layer and so will reduce the overtopping performance. This represents a key technical risk associated with the design.

### 5.4 Tie-ins with existing defence

The tie-ins have been considered at a conceptual level but will require careful consideration during detailed design.

### 5.5 Pavement closure during a storm event

The higher tolerable thresholds offered by the wall mean that the pavement area behind the defence will be unsuitable for public pedestrian access during the regular storm event (1 in 1-year). Consequently, DoI should provide emergency on-the-ground manpower during this storm event, to cordon off and close parts of the frontage to reduce the risk of public interaction with wave overtopping. This should be factored when considering the suitability of each defence option.

### 5.6 Services

No services information has been provided as part of this study. If the project progresses to outline and detailed design it will be essential that a full service plan is developed.

### 5.7 Construction accessibility

Prior to the development of outline designs it would be advisable to appoint a construction contractor to

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Client Department of Infrastructure, Isle of Man Government  
Day, Date and Time 26/08/2014  
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Subject Peel Open Coast – Option POC1 – Rock Revetment



provide constructability advice. Although the site is considered reasonably accessible it would be beneficial to confirm the proposed methods of construction and temporary works required.

## 5.8 Stakeholder requirements

A Multi Criteria Analysis was completed as part of this study to try and determine the key considerations of the project stakeholders. It is anticipated that during the course of a formal options appraisal project stage that more in depth stakeholder consultation will be completed. The results of which may lead to changes in the concept designs that have already been developed.

## 5.9 Environmental impacts

No formal Environmental Impact Assessment was completed during this project stage. It is anticipated that during the course of an options appraisal stage that an in depth assessment of the environmental impacts associated with all proposed options would be considered. This process may result in changes being made to the proposed designs.